EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS
Earthquake Engng. Struct. Dyn. **28**, 1061–1065 (1999)

INFLUENCE OF MASONRY INFILL PANELS ON THE VIBRATION AND STIFFNESS CHARACTERISTICS OF R/C FRAME BUILDINGS

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SUMMARY

Vibration measurements were performed on two adjacent, three-storey reinforced concrete frame buildings with hollow clay brick infill panels. The first building was a bare frame, and the second one was a similar frame infilled with brick panels. The fundamental period for the infilled frame building was much smaller than that of the bare frame building. Using shear beam lumped mass models and the vibration data, the actual lateral stiffness of both buildings was identified. The lateral stiffness of the infilled frame building was found to be seven times that of the bare frame building. Four numerical models of the infilled frame building were constructed. The frame and floors were represented using an experimentally validated model, and the infill panels by one of three commonly used 'equivalent diagonal truss' models or by plane stress finite elements. Only the plane stress finite element model produced a reasonable agreement with the experimental results. Copyright © 1999 John Wiley & Sons, Ltd.

KEY WORDS: masonry infill; infilled frame; initial stiffness; numerical models; ambient vibrations; small strain

1. INTRODUCTION

Reinforced concrete frames with hollow clay brick masonry infill panels are very common in many parts of the world. The earthquake response of such structures has received considerable attention worldwide. A number of researchers have studied the stiffness[1] and strength characteristics of the infill panels under in-plane reversed cyclic loading up to failure. More recently, comprehensive analytical models[2,3] have been proposed for predicting the hysteretic behaviour of the panels including strength and stiffness degradation, pinching slippage and unilateral contact. The scope of this paper is limited to the influence of the panels on the initial in-plane building stiffness at small strains, and the suitability of some models[4] commonly used to represent the infill panels for predicting the dynamic properties of such structures.

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CCC 0098–8847/99/091061–05$17·50

Received 12 July 1997
Revised 5 March 1998 and 1 April 1999
Accepted 7 April 1999

2. EXPERIMENTATION

Two adjacent, three-level R/C frame buildings of identical design, located in Rouiba, an eastern suburb of Algiers, Algeria, were selected for a side-by-side comparison. Both apartment buildings were essentially symmetrical in plan, regular in elevation (Figure 1) and designed according to the 1983 Algerian earthquake-resistant building code.[5] Stiff, 1-m high reinforced concrete shear walls were cast in place along the perimeter of the sanitary crawl space of both buildings. The foundations and soil conditions were identical. For such buildings, soil–structure interaction is not expected to have a significant influence, and any difference in the structural response of the two buildings can be attributed to the masonry infill panels. When vibration measurements were performed, the first building had only the frame and floors, and the second had a similar frame infilled with brick panels for the walls and partitions, and floors.[6]

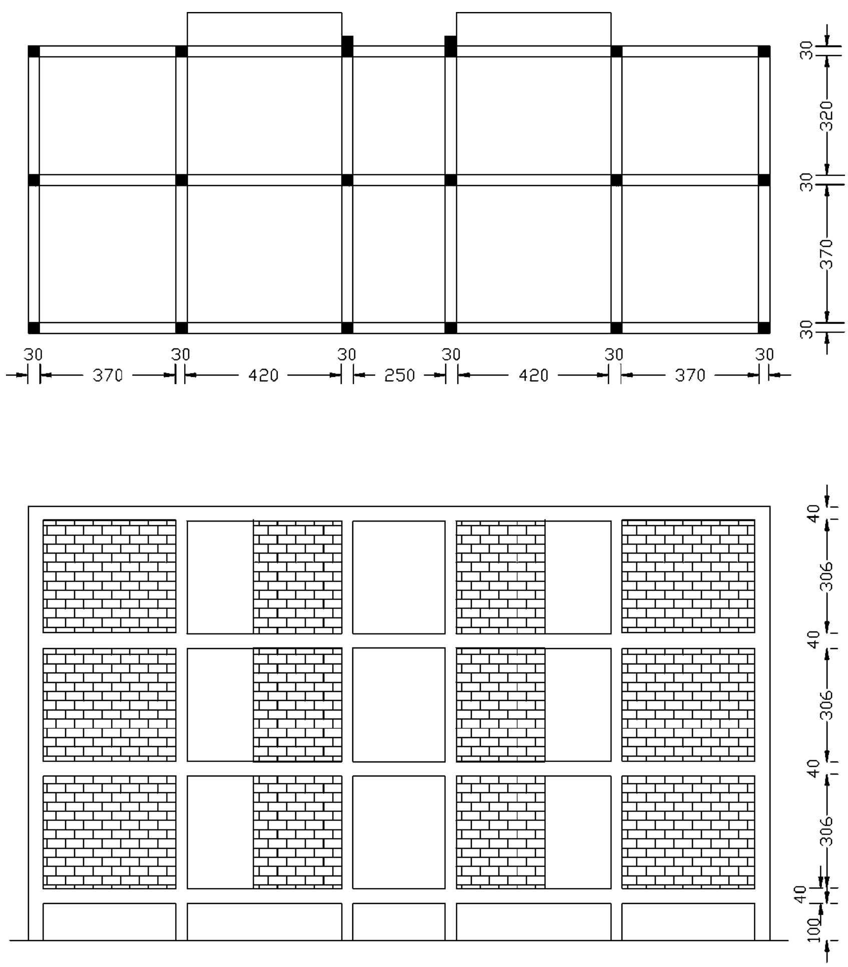

Figure 1. Typical floor plan and infilled frame (dimensions in cm)

Table I. Frequencies and mode types for the bare frame and the infilled frame buildings

Bare frame building		Infilled frame building	
Frequency (Hz)	Displacement pattern	Frequency (Hz)	Displacement pattern
2·500	Transverse translation	5·750	Transverse translation
2·875	Longitudinal translation	6·875	Longitudinal translation
2·875	Torsion	9·125	Torsion

The response of these two buildings to wind-induced vibrations was recorded using Bruel & Kjaer equipment suitable for very low levels of acceleration. Although the wind velocity and pressure, and the frequency content of the excitation were not known with accuracy, the usual hypotheses for analysing the recorded motion were assumed to hold, i.e. that the excitation was stationary, ergodic and could be considered a white noise. The information on the frequency content of the response was extracted by Fourier analysis of the auto-correlation function of the recorded signal, and identifying the peaks of the power spectral density thus obtained. Table I gives the frequencies of the first three modes and shows a markedly higher fundamental frequency and therefore stiffness for the infilled frame building, pointing to the fallacy of treating these infill panels as 'non-structural' elements.

3. SYSTEM IDENTIFICATION

The stiffness properties of the two buildings were identified from their mass and vibration characteristics using simple shear beam lumped mass models, where only the horizontal translation degrees of freedom at each floor were retained. Since the circular frequency ω_1 for the first transverse translation mode was determined experimentally with a high degree of confidence, the characteristic equation of the three d.o.f. model becomes, for that value ω_1, a third degree equation with only one unknown, the effective story stiffness in the transverse direction k. The smallest root is retained as the correct value for the story stiffness, since it allows the potential energy to be minimum. The lateral storey stiffness of the infilled frame building (49608·4 kN/m) was found to be 7·0 times that of the bare frame building (7042·3 kN/m), a result which allows to quantify the lateral stiffness contributed by the infill panels.

4. NUMERICAL MODELS

The capability of four models of the infill panels to predict the observed dynamic behaviour was then investigated using numerical models. Eigenvalue analyses were performed to obtain the vibration frequencies for the different models. First, a model was developed for the frame and floors, the results of which (first nine frequencies) were in very good agreement with the experimental data for the bare frame building. Using that model for the frame and floors, four distinct numerical models for the infilled frame building were developed. The infill panels were represented in three models by diagonal truss elements[7] with width determined according to

Table II. Frequencies of numerical models vs. experimental frequencies (Hz)

Modes	Diagonal truss models			Plane stress model	Experimental results
	Tahar[18]	Thiruvengadam[21]	Ciongradi[22]		
1st Long.	3·723	3·489	3·738	5·019	6·875
1st Transv.	4·731	4·097	4·781	5·750	5·750
1st Torsion	5·923	5·082	5·995	7·219	9·125

the formulae proposed by Tahar,[8] Thiruvengadam,[9] and Ciongradi,[10] and by plane stress isoparametric quadrilateral finite elements in the fourth. Table II compares the frequencies obtained for the four numerical models with the experimentally determined frequencies and shows that for predicting the frequencies of the first few modes under small strain conditions the diagonal truss models are not appropriate, while the plane stress model is reasonably accurate.

That the diagonal truss models could not correctly predict small strain behaviour is not surprising in view of the fact that they are based on the following assumptions:[4] (i) the horizontal shear and the deformation of the structure are large enough to cause loss of contact between the panel and the columns in two diagonally opposite corners; (ii) the state of stress in the masonry panel causes cracks to appear in the bricks and/or the mortar, delineating a diagonal truss element between the other two corners. Furthermore, the force and displacement boundary conditions of an isolated panel/frame assembly differ from those of a panel in a complete structure.[11]

5. CONCLUSION

The dynamic tests on two actual structures demonstrated and quantified the stiffening effects of the infill panels at small strain levels. Based on the experimental results it was found that plane stress finite elements provided a better representation of the in-plane initial (small strain) stiffness of the infill panels.

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